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MATERIALS INFORMATION

ENGINEERING MATERIALS OFFICE

PAVEMENT RUTTING INVESTIGATION

HIGHWAY 401; PORT HOPE DISTRICT

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ABSTRACT

Severe rutting has been observed on the eastbound driving lane of Hwy. 401 between Oshawa and Trenton. A site investigation was conducted to determine the cause of the rutting problem.

The investigation involved sawing and removing a small strip of the asphalt pavement, coring of other locations of the highway, examining and sampling of granular base and subbase materials and performing laboratory evaluation. Surface profiles of sampled locations were also measured.

The results indicate that the cause of the rutting problem is due to the inadequate mix stability of the HL1 and HL4 resurfacing courses. The original hot mix pavement, base and subbase were found to be adequate.

ACKNOWLEDGEMENTS

This investigation represents a combined effort by MTC staff from the Central Region and the EMO. The writer would like to thank K. McClelland of Port Hope District Maintenance Office for arranging the traffic control, M. MacLean of the Soils and Aggregates Section for preparing the base and subbase investigation report, J. Graham, R. Oldford and J. Pretty of Bituminous Evaluation Laboratory for the sampling and testing. Photographs of laboratory samples were taken by the Audio Visual Office staff and the typing was done by Nina DeRocchis.

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Appendix 1 - Soil Investigation Report

2 - Bituminous Mix Design Results
- Related Memorandum

3 - Surface Condition Survey

1. Introduction

Port Hope District Maintenance staff recognized a severe rutting problem on the eastbound driving lane of the Hwy. 401 between Oshawa and Trenton. The rutting was so severe that six small areas between Hwys. 28 and 45 had to be treated immediately (in October 1982).

The Bituminous Section was informed of the problem and with the permission and co-operation of the Port Hope District, Maintenance staff, a site investigation was conducted on 82 09 22.

The investigation involved sawing and removing a small strip of the asphalt pavement to enable the granular base and subbase to be examined. In addition to sampling the asphalt pavement and granular materials from the strip, coring from other locations of the highway was done for laboratory evaluation.

2. Aim of Site Investigation

The aim of the investigation is to determine the cause of the rutting by:

(a) Visual examination of the different layers of the pavement.

(b) Determination of the material properties of different asphalt pavement layers and base and subbase materials.

(c) Measuring the surface rutting profiles of the sampled areas.

3. Site Location and Contract Information

The section of Hwy. 401 involved in this investigation is on the eastbound driving lane between Hwys. 28 and 45. Samples were obtained from the following areas:

	Location	Distance from Hwy. 28
(A) sawing	-	3.6 km
(B) coring	1	2.5 km
	2	3.6 km
	3	3.8 km

Based on the 1980 traffic record, the traffic on this section of Hwy. 401 was 23,000 AADT for both directions. The directional split as recorded by the Research and Development Branch in 1982 was 49% westerly and 51% easterly with 25% heavy vehicles, i.e. some 3,000 trucks per day in the eastbound direction.

The pavement was constructed under two different contracts i.e. 60-92 and 74-167. The first contract involved laying a 102 mm HL8 binder course and a 32 mm HL1 surface course. The pavement was then resurfaced in 1975 using a HL2 leveling course, HL4 binder course and HL1 surface course. The total hot mix thickness for the pavement is 230 mm (9 in). The base and subbase thicknesses were 152 mm (6 in) of granular type 'A' and 380-457 mm (15-18 in) of granular type 'B' material, respectively. A summary of information on the hot mixes for the two contracts are listed in Table 1 and 5.

4. Site Investigation

On 82 09 22, a full scale site investigation was carried out on the section of Hwy. 401, 25 km east of Hwy. 28. Traffic was blocked off for 2 km to cover the areas for coring and sawing (Figure 1). The following describes the three types of sampling operations:

(a) Coring - Four cores were taken from each of the three locations as mentioned in (3.) above as follows:

core 1 at 3.35 m (11 ft.) from the edge of the pavement (i.e. 0.3 m from centre line).
core 2 in the left wheel track (LWT)
core 3 between the wheel tracks
core 4 in the right wheel track (RWT)

After coring, the transverse profile was measured with a straight edge placed across the driving lane of the cored area. The distance between the road surface and the bottom of the straight edge (the datum) at different intervals is reported in TABLE 2.

(b) Sawing - A strip 0.61 m (2 ft.) wide by 3.35 m (11 ft.) long (i.e. 0.3 m short of a full lane width) and 254 mm (10 in.) deep (i.e. full depth of the pavement) was cut by a power saw (Figure 2). The strip was then cut into blocks for easy removal of the pavement materials (Figures 3 and 4). After the removal of the blocks, the profile of the road surface was measured (Figures 6 and 7). The profiles measured insitu were used as reference for further measurements of different layer thickness in the laboratory (TABLE 4). The crossfall at this point was recorded as 2.5%.

(c) Base and subbase investigation - This consisted of:

(i) field examination of base and subbase materials exposed after cutting of the strip (Figure 8)

(ii) sampling of insitu materials for further laboratory testing from a test hole located 2 m from the edge of the pavement. The investigation was carried out by the Soils and Aggregates Section.

5. Laboratory Evaluation

(a) Asphalt Pavement Samples - A total of 16 cores (full depth) was taken from the three locations. The thickness of each layer was measured and catalogued for each of the cores (TABLE 3). The different layers of each core were separated by sawing and the following tests were performed:

i) gradation
ii) % asphalt cement
iii) penetration and viscosity of the recovered A.C.
iv) Marshall properties (on recompacted samples)
v) % compaction and % voids in pavement
vi) % crushed of the HL4 and HL8 coarse aggregates

The blocks removed from the sawed strip were reassembled in the laboratory and measurements of different layer thicknesses were made at various intervals across the width of strip. The data (TABLE 4) were plotted in Figure 10.

(b) Base and Subbase Samples - The base and subbase samples were tested for the following:

i) moisture content
ii) Proctor maximum density
iii) gradation
iv) % crushed particles
v) California Bearing Ratio

6. Results and Discussion

(a) Surface Profiles - The transverse surface profiles of each of the three cored sections and the sawed areas were measured and are plotted in Figures 9 and 10.

The profiles generally show similar rutting along the wheel tracks. The R W T rutting was more severe than the L W T. The double ruts (Figures 5 and 7) occurring in the same wheel track

are apparently caused by the dual wheels of trucks. This is a good indication that the asphalt mixtures were sufficiently plastic to deform to the shape of the twin wheels. The greater deformation of the R W T is probably due to the shift of the centre of gravity of the truck travelling on the road surface with a crossfall, causing a heavier load on the right side of the axle. A rut depth (i.e. peak to valley height) of some 32 mm (1.26 in) was recorded in places (Figures 9 and 10).

(b) Layer Thicknesses - The pavement consists of a total of 230 mm (9 in.) of hot mix from two different contracts as shown in Table 1. The different layer thicknesses as measured from the cores and the sawed blocks are compared with specification. The layer profiles and thicknesses show that the rutting occurred only in the top two lifts i.e. the HL1 surface course and HL4 binder course placed in 1975. Therefore, more discussion will be made on these two courses than the others layers as follows:

(i) HL1 - In looking at the core samples, the HL1 thickness in both the wheel tracks, was thinner than that near the centre line and between the wheel tracks. The specified thickness for the HL1 was 38 mm, whereas, an average thickness of 27 mm was recorded (ranging from 5 mm to 45 mm, Table 3). At the R W T of location 1, a thickness of 5 mm was measured. This thickness is even less than the nominal stone size of 9.5 mm. It was noticed that the HL1 stone had been slightly bedded in the HL4 binder course.

It is noted that the HL1 at location 1 and 2, being thinner than the specified thickness, had undergone more rutting than location 3 (Figure 9) where the thickness was close to the specificaton.

The very thin layer thickness of the HL1 again showed up in the sawed strip as seen in Figure 10. It is clear that the HL1 was pushed towards the right side of the wheel tracks, i.e. the lower end of the crossfall in the direction of the shoulder. The HL1 at the "pushed-up" area was therefore thicker than the rest of the cross-section. However, the thickness was still less than the specified (i.e. 25 mm versus 38 mm). The HL1 had been laid too thin originally.

(ii) HL4 - The thickness measurments of the HL4 from the cores show that they were either near to or thicker than the specified 44 mm. The range is between 42 and 70 mm. The sawed strip HL4 layer appears to be thicker than specified, i.e. average 65 mm versus 44 mm and the thickness at the RWT was 70 mm.

(iii) HL2 - The thickness varies between 13 and 30 mm. Since the mix is used as levelling course, the variation is as expected.

(iv) Old HL1 - On average, the thickness of the old HL1 was very close to what was specified (32 mm). As seen from the picture (Figure 7), the layer thickness seems to be very consistent ranging from 25 mm to 33 mm.

(v) Old HL8 - The HL8 was laid in two lifts. The average layer thicknesses obtained in location 1 and 2 were thinner than specified (102 mm), whereas, for location 3 and the sawn strip, the thickness was close to specified.

(c) Material Properties. The material properties obtained from the different tests are compared with the original mix designs and they are discussed as follows:

(i) Aggregate Gradations

The gradation obtained for the HL1 and HL4 are very close to the mix designs (Figure 12, 13). Both the core samples and mix design gradations are also close to the Fuller maximum density curves, i.e. these two mixes are 'tight' or dense.

(ii) % Asphalt Cement

The A.C. contents obtained are between 0.4 and 0.9 percent less than the mix design values for all of the mixes (Tables 5 and 6). The lower values could be partly due to the low original A.C. contents as well as the unextractable A.C. absorbed by the aggregates, because of the age of the pavement.

(iii) Marshall Properties on Recompacted Samples

% Voids - The void contents (Table 6) obtained for the top two courses (i.e.the HL1 & 4) are below the current specification of 3%. This indicates that insufficient void space is available to accommodate the A.C. under the traffic compaction of the mixes. This is evident by looking at the samples from the sawn strip that A.C. is migrating (or bleeding) through to the surface: hairline traces of the 'bleeding' A.C. are visible on the surface (Figure 12).

Marshall Flow and Stability - The Marshall stability from the recompacted core samples for the HL1 and HL4 are 17,460 N and 11,530 N respectively (TABLE 6). The flow value (15.6) of the HL1 is slightly high and the HL4 has a value of 12.2. The HL8 mix has a very high flow value (24.2) and a stability of 9,600 N. This mix appears 'tender' but adequate for the type of traffic before it was resurfaced in 1975 (about 18,000 A.A.D.T. for both directions).

(iv) Pavement Voids and Compaction

The % voids in the pavement are 3.8% and 3.9% for the HL1 and HL4 respectively. These values are close to what normally expected from these mixes. The % compaction of these two mixes is 98.5% i.e. near to full compaction. The following compares the % voids and % compaction of pavement samples taken from the R W T and a location 0.3 m from the centreline for the different mixes.

MIX	0.3 m from Centreline		R W T	
	% Voids	% Compaction	% Voids	% Compaction
HL1	4.4	98.2	3.2	98.8
HL4	5.4	97.2	2.4	99.8
Old HL1	3.3	98.8	3.6	99.5
HL8	2.8	98.5	3.2	98.5

Assuming there is no traffic compaction in the area of 0.3 m from the centreline it can be seen that the HL4 has densified considerably from 97.2% to 99.8% with a decrease in % voids from 5.4% to 2.4%. The HL1 mixes (both old and new) have also densified by about 0.6% compaction whereas the HL8 % compaction remains the same. The slightly higher % voids at the R W T compared with that near the centre line for the old HL1 and HL8 may be due to lower original compaction.

(v) Aggregate Characteristics

The aggregate types of different mixes are shown in Table 1. The coarse and fine aggregates for the HL4 mix are Nelson gravel and Nelson screenings and sand. The coarse aggregate has about 60% crushed particles. This percentage is considered low for a mix that is required to produce high stability. Fig.15 shows the distribution of the rounded gravel in the matrix of the mix. The fine aggregates used in the HL1 and 4 mixes is also rounded beach sand. It is generally recognized that the more crushed aggregate in a mix, the higher the stability obtained. This is due to the higher interparticle friction and aggregate interlocking properties in enhancing the resistance to shearing movement between aggregate particles. In comparing the matrices of the HL1 (Fig.16) and the HL4 (Fig.15), less aggregate interlock is observed in the HL4 then the HL1 e.g. coarse aggregate is more separated by sand/A.C. mortar. This together with the rounded aggregate shape has lead to observed plasticity and deformation/rutting of the HL4 mix under the compaction of the traffic.

(vi) Base and Subbase Material Properties

An investigation report prepared by the Soils and Aggregates Section is included in Appendix 1. The results show that the materials conform to the relevant MTC specifications and the moisture tests confirm that the pavement base and subbase were dry. However, the % crushed of granular particle was found sub-standard, i.e. 39% as compared to 50% minimum. The CBR (standard compaction) value was marginal, i.e. near to the minimum 80% suggested requirement.

(d) Mix Design. The mix design properties i.e. stability, V.M.A. and void content of the different mixes are discussed below. It revealed that the HL1 and 4 mix designs for this section of highway are apparently inadequate. Contract information shows that the low stability and V.M.A. properties were recognized and had been discussed (App. 2):

(i) Stability and flow - The mix design stability of 5600 and 4900 N for the HL1 and HL4 (Table 5) are too low as compared with the specification requirements (then asking for 7117 N

(1600 lb) and 6227 N (1400 lb) respectively). The mix design stability for the old HL1 and 8 seem to be sufficient and performing as no deformation is observed. The flow values for all the mixes are close to the mix design and specification requirement.

(ii) V.M.A. - The mix design values for the HL1 and 4 are 14.8% and 14.2% respectively, and are below the specified minimum of 15.0% (Table 5). The same is applied to all the plant check values, especially after allowing the correction for A.C. absorption (except for the old HL1 mix which has a plant check value of 16.5%).

On the whole, the V.M.A. appears to be insufficient to accommodate the volume of A.C. in the HL1 and 4 mixes resulting in overfilling the void space between the aggregates. This would lead to bleeding (Fig.12) and rutting/pushing of the mixes under the compaction of the traffic. This is what had happened in this section of the pavement.

(iii) Void content - The % voids for the HL1 and 4 are 2.7% and 3.0% respectively, although meet the old specification minimum of 2%, but the plant check values are low (i.e. 1.8% and 2.0% respectively for the HL1 & 4). Again, this encourages flushing/bleeding, and indeed the construction report mentioned that the HL1 mix had the 'slick' appearance in the wheel tracks shortly after construction. This is a sign of a low void content mix which is unsuitable for use in a high traffic volume pavement. The other mixes (i.e. old HL1 & 8) also had low voids as seen from the plant checks, especially the HL8 mix (% voids = 1.0%).

7. Conclusions

1. The cause of the rutting problem is the inadequate mix stability and V.M.A. in the HL1 and 4 resurfacing courses, particularly the HL4 mix. The mix designs did not meet specification requirements.

2. The core samples from the rutted areas show that HL1 thickness is thin with a thick but unstable HL4 mix underneath. Even when the HL1 was laid to the specified thickness, rutting still occurred.

3. The old pavement layers from the first contract appear to be in good condition and are not seen to be the cause of the rutting problem.

4. The base and subbase are considered adequate for the pavement.

8. Recommendations

1. Since the cause of the rutting is the poor stability property of the HL1 and HL4 mixes, these materials have to be removed in any future remedial work. (If a new surface layer were to be laid on top without removing the HL4, the rutting would reappear). A thickness of about 25 mm of the HL4 could be left without removal.

2. New mixes for resurfacing work should be designed to meet specification requirements.

3. A complete pavement condition survey of the remainder of Hwy.401 between Oshawa and Trenton should be carried out since apart from the rutting, multiple and/or alligator crackings have been observed on the pavement (App. 3).

4. The pavement condition survey should also extend to other major highways (e.g. Hwy. 400) which show signs of rutting.

5. A laboratory research program should be set up to study the adequacy of mixes for use on high density/volume traffic highways, and to establish criteria for mix designs for such types of highway traffic.

Contract	Mix Type	Thickness mm (in.)	Coarse Aggregate	Fine Aggregate	C:F (%)	Design A.C.(%)
74-167	HL-1	38 (1- 1/2)	Havelock Traprock	Nelson Pit Scr. + Sand	45:55	5.2
	HL-4	44 (1- 3/4)	Nelson Pit	Nelson Pit Scr. + Sand	40:60	5.1
	HL-2	19 (3/4)	-	Nelson Pit Scr. + Sand	-	6.9
60-92	HL-1	32 (1- 1/4)	Havelock Traprock	Jibb Pit, Delong + McHolme (Bl.Sa.)	45:55	5.6
	HL-8	102 (4)	Jibb Pit		55:45	5.4

TABLE 1: Hot mix types, thicknesses and materials of the pavement.

TABLE 2: PROFILE MEASUREMENTS OF CORED LOCATIONS

LOCATION 1		LOCATION 2		LOCATION 3	
Distance From Edge of Road (m)	Reading From Datum (mm)	Distance From Edge of Road (m)	Reading From Datum (mm)	Distance From Edge of Road (m)	Reading From Datum (mm)
0	78	0	80	0.30	70
0.24	75	0.30	72	0.55	65
0.40	70	0.40	67	0.73	77
0.52	80	0.52	80	0.91	73
0.61	84	0.61	90	1.13	77
0.79	80	0.73	90	1.22	70
0.91	80	0.82	85	1.34	67
1.22	75	1.00	88	1.49	60
1.37	68	1.07	88	1.65	60
1.52	65	1.22	75	1.83	63
1.83	67	1.37	70	2.13	63
1.92	63	1.52	63	2.44	63
2.13	55	1.74	63	2.56	70
2.26	60	1.83	65	2.74	70
2.44	78	2.01	53	2.90	74
2.74	80	2.13	57	3.14	74
3.05	75	2.32	70	3.35	75
3.23	72	2.44	83		
3.35	73	2.62	80		
		2.80	85		
		3.02	75		
		3.35	75		
Crossfall = 3%		Crossfall = 2%		Crossfall = 2.5%	

MIX	LOCATION 1				LOCATION 2				LOCATION 3				SPEC.
	1	2	3	4	1	2	3	4	1	2	3	4	
HL-1	25	13	12	5	28	25	35	30	45	35	36	30	38
HL-4	48	55	70	62	50	57	70	58	55	55	55	42	44
HL-2	28	23	17	19	30	25	20	23	23	27	13	23	19
HL-1	34	32	30	24	32	30	32	30	33	26	33	26	32
HL-8	87	87	88	97	87	97	100	105	100	100	105	98	102

Core 1 - 0.3 m From Centre Line.
2 - Left Wheel Track (L.W.T.)
3 - ℄ of Driving Lane
4 - Right Wheel Track (R.W.T.)

TABLE 3: LAYER THICKNESSES (mm) OF DIFFERENT MIXES OF THE CORED LOCATIONS

TABLE 4

LAYER THICKNESS OF PAVEMENT AT SAWED STRIP

Distance from Edge of Road (m)	Profile Measurement (from straight edge to HL1 surface) (mm)	Layer Thickness (mm)				
		Contract 74-167			Contract 60-92	
		HL1	HL4	HL2	HL1	HL8
0	50	33	78	10	29	100
0.24	33	26	89	15	25	110
0.35	32	24	86	15	30	115
0.49	55	19	68	20	27	106
0.56	57	17	70	21	27	110
0.76	50	17	69	21	31	107
0.85	53	15	70	18	30	107
0.90	53	10	73	27	20	110
1.11	40	12	74	20	28	101
1.22	35	10	78	19	26	107
1.31	30	12	76	19	27	106
1.43	30	10	74	21	25	110
1.52	33	15	73	18	26	108
1.71	31	12	71	16	28	108
1.84	30	10	70	18	28	109
1.86	26	13	72	15	32	98
2.11	21	13	77	19	31	95
2.19	24	11	76	31	29	98
2.26	31	7	78	13	32	95
2.32	40	5	62	23	30	95
2.38	45	5	58	24	30	98
2.44	43	7	58	22	33	95
2.53	40	8	61	26	28	97
2.59	40	12	56	27	30	95
2.68	45	8	55	27	25	100
2.74	45	9	56	25	28	97
2.87	40	8	59	26	30	97
2.96	40	10	60	27	31	92
3.11	42	8	56	26	33	92
3.35	42	9	56	30	30	95

Crossfall = 2.5%

TABLE 5

MIX DESIGN AND PLANT CHECKS PROPERTIES

Mix Type	MARSHALL PROPERTIES					% RETAINED CUMULATIVE										
	% A.C.	% Voids	Flow	Stab. (N)	V.M.A. (%)	16.0 mm	13.2 mm	9.5 mm	6.7 mm	4.75 mm	2.36 mm	1.18 mm	600 um	300 um	150 um	75 um
HL-1 Mix Design	5.2	2.7 (2)	9.2 (9)	5,649 (7,117)	14.8 (15.0)			21	38	45	52	61	73	88	96	98.5
HL-1 Plant Checks	5.17	1.8	10.5	11,600	15.0*					46						
HL-4 Mix Design	5.1	3.0 (2)	9.5 (9)	4,893 (6,227)	14.2 (15.0)		5	23	33	40	50	61	71	84	96	98.3
HL-4 Plant Checks	5.05	2.0	10.0	8,870	14.1*					42						
HL-2 Mix Design	6.9	2.8 (3)	9.3 (9)	4,003 (3,558)	18.0 (18.0)			1	4	7.7	24	41	56	75	92	96.1
Old HL-1 Mix Design	5.6	(2)	(9)	(7,117)	(15.0)					45						
Old HL-1 Plant Checks	5.4	3.0	9.9	8,827	16.5*					48						97.6
HL-8 Mix Design	5.4	(2)	(9)	(6,227)	(14.5)					55						
HL-8 Plant Checks	5.1	1.0	12.6	6,712	13.6*					53						96.4

( ) = specification minimum

* = uncorrected for A.C. Absorption

Above Marshall stability obtained using the old heads and could be increased by 1300 N if tested in new heads.

TABLE 6

CORE SAMPLES PROPERTIES

Mix Type	ABSON	MARSHALL RECOMPACTION			PAVEMENT SAMPLE			% RETAINED CUMULATIVE										
	% A.C.	% VOIDS	FLOW	STAB. (N)	% VOIDS	% COMP.	LIFT	16.0 mm	13.2 mm	9.5 mm	6.7 mm	4.75 mm	2.36 mm	1.18 mm	600 um	300 um	150 um	75 um
HL-1	4.78	2.4	15.6	17,460	3.8	98.5	Surface			16	36	47	59	68	78	87	95	97.6
HL-4	4.70	2.5	12.2	11,530	3.9	98.5	Binder		5	23	35	43	54	64	73	84	95	97.3
HL-2	6.47	3.5	12.6	9,640	7.5	95.9	Level-ling				1	6	26	43	57	73	90	94.5
Old HL-1	4.86	3.5	13.8	19,020	3.8	99.2	Old Surface			14	37	48	60	65	70	80	93	97.1
HL-8	4.53	1.6	24.2	9,660	3.0	98.5	Old Binder	15	25	38	50	57	67	74	80	87	93	96

Fig. 1: Traffic Control of Sampling Area (Looking Westerly, Note Water Ponding on the Wheel Track Ruts)

Fig. 2: Sawing of the Pavement

Fig. 3: Removing the Sawed Pavement Samples

Fig. 4: Pavement Samples

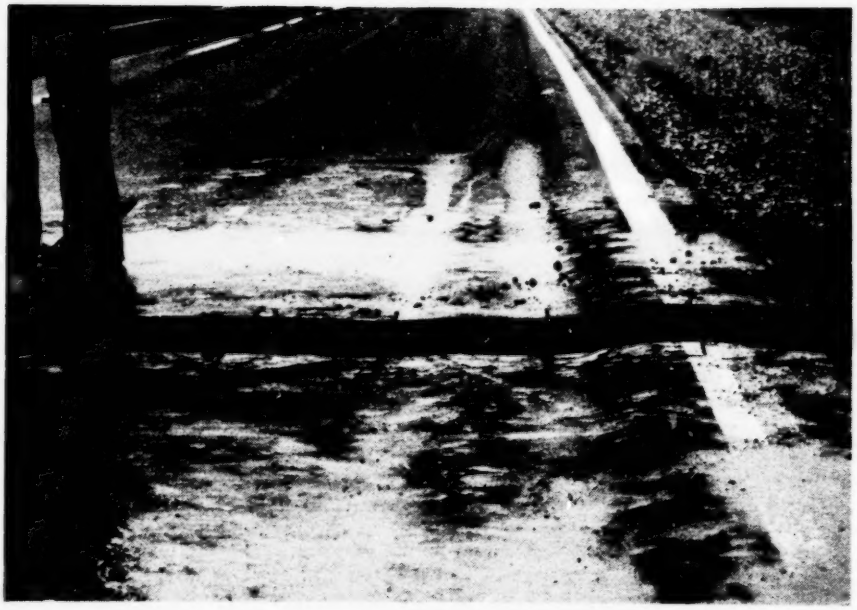


Fig. 5: Trench After Removal of Pavement Materials

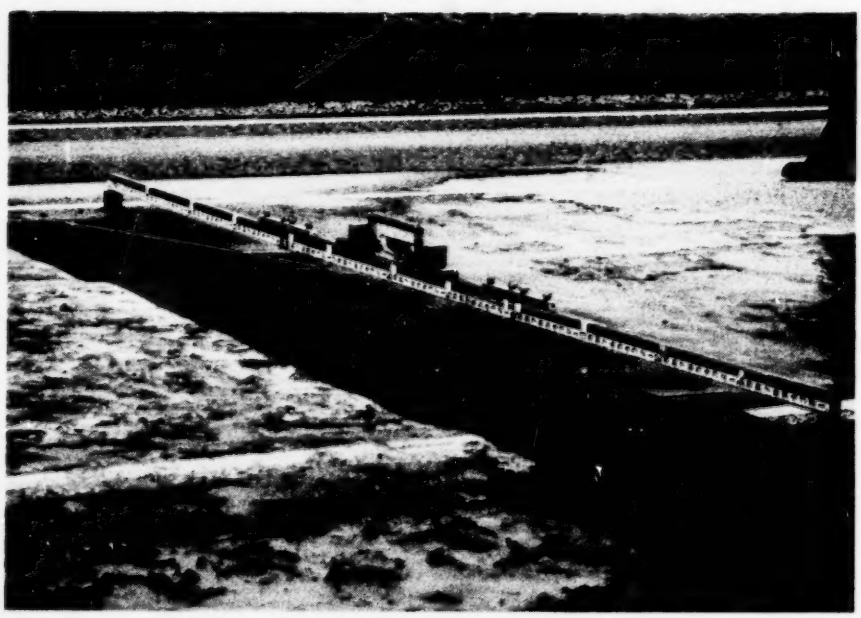


Fig. 6: Profile Measurement

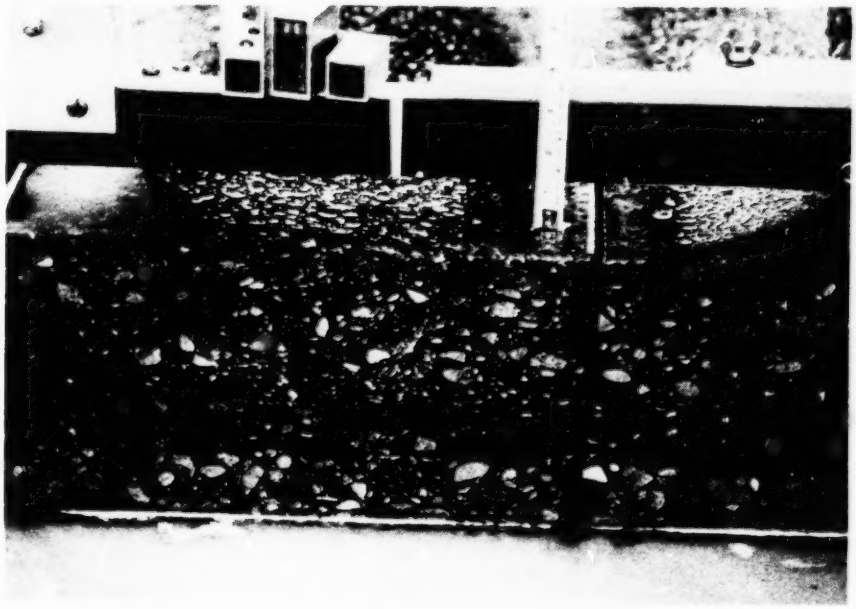


Fig. 7: Close-up of R.W.T. Rutting

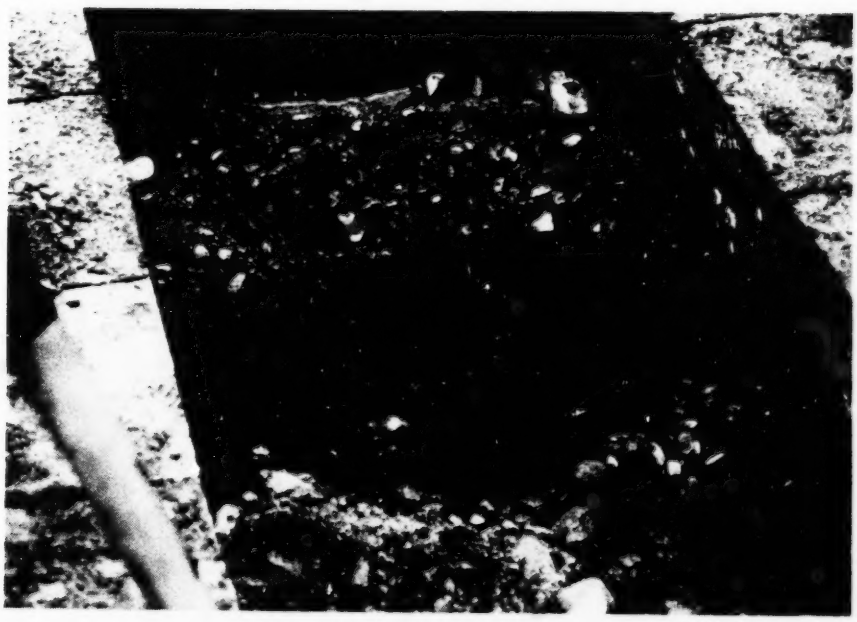


Fig. 8: Condition of the Granular Base Materials

FIG. 9 SURFACE PROFILES OF CORED AREAS

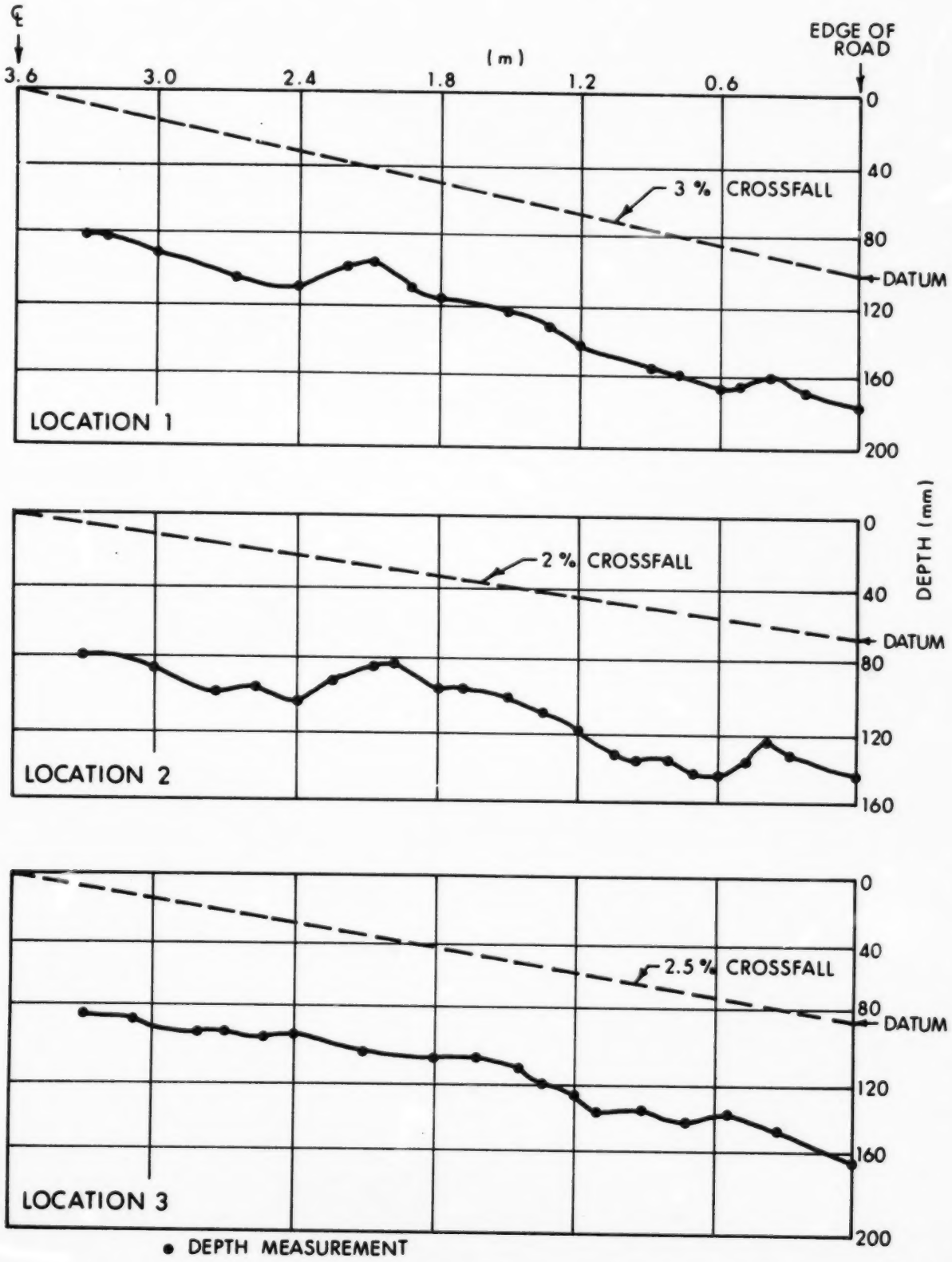
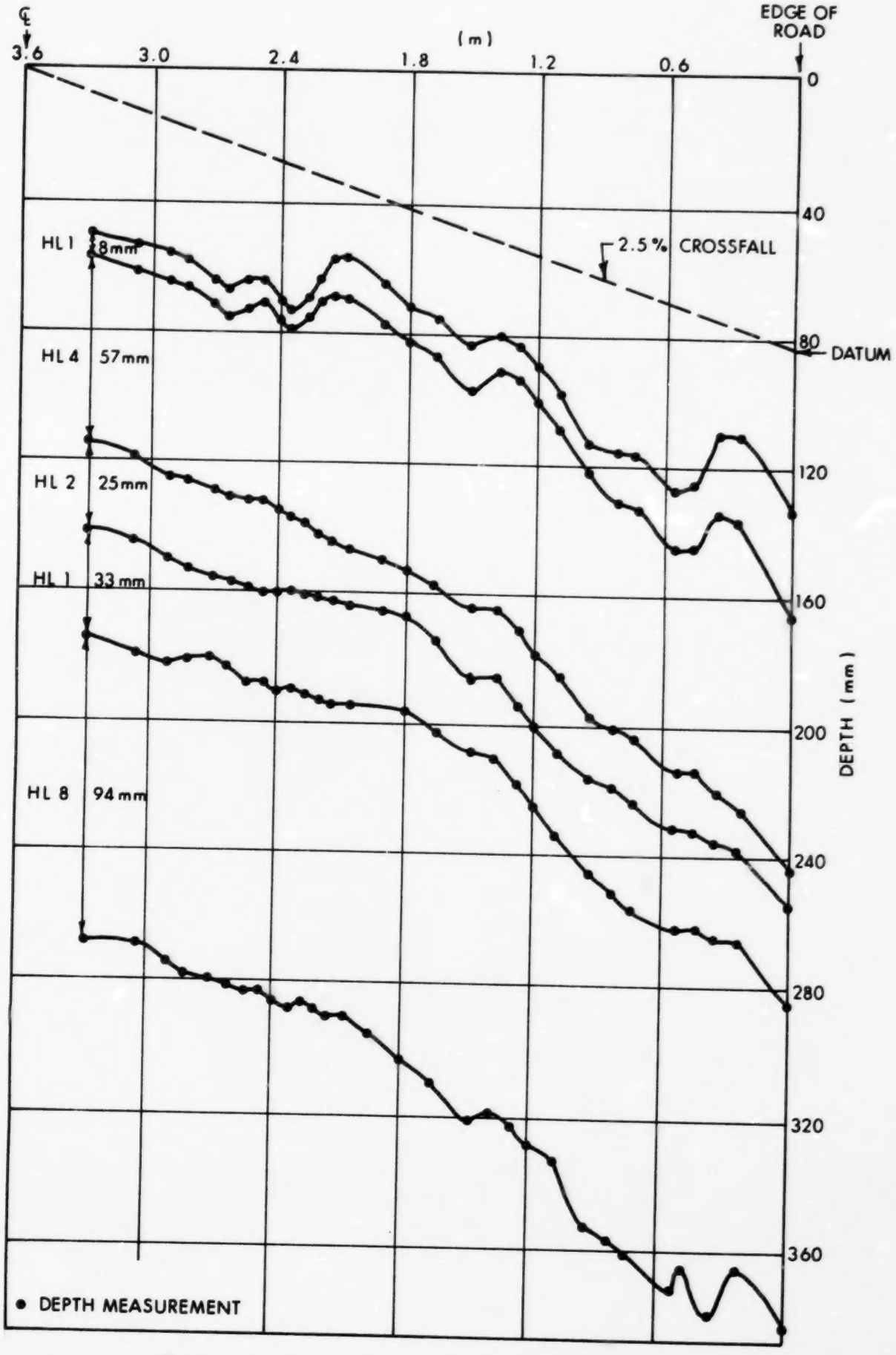


FIG. 10 LAYER THICKNESS OF PAVEMENT SECTION



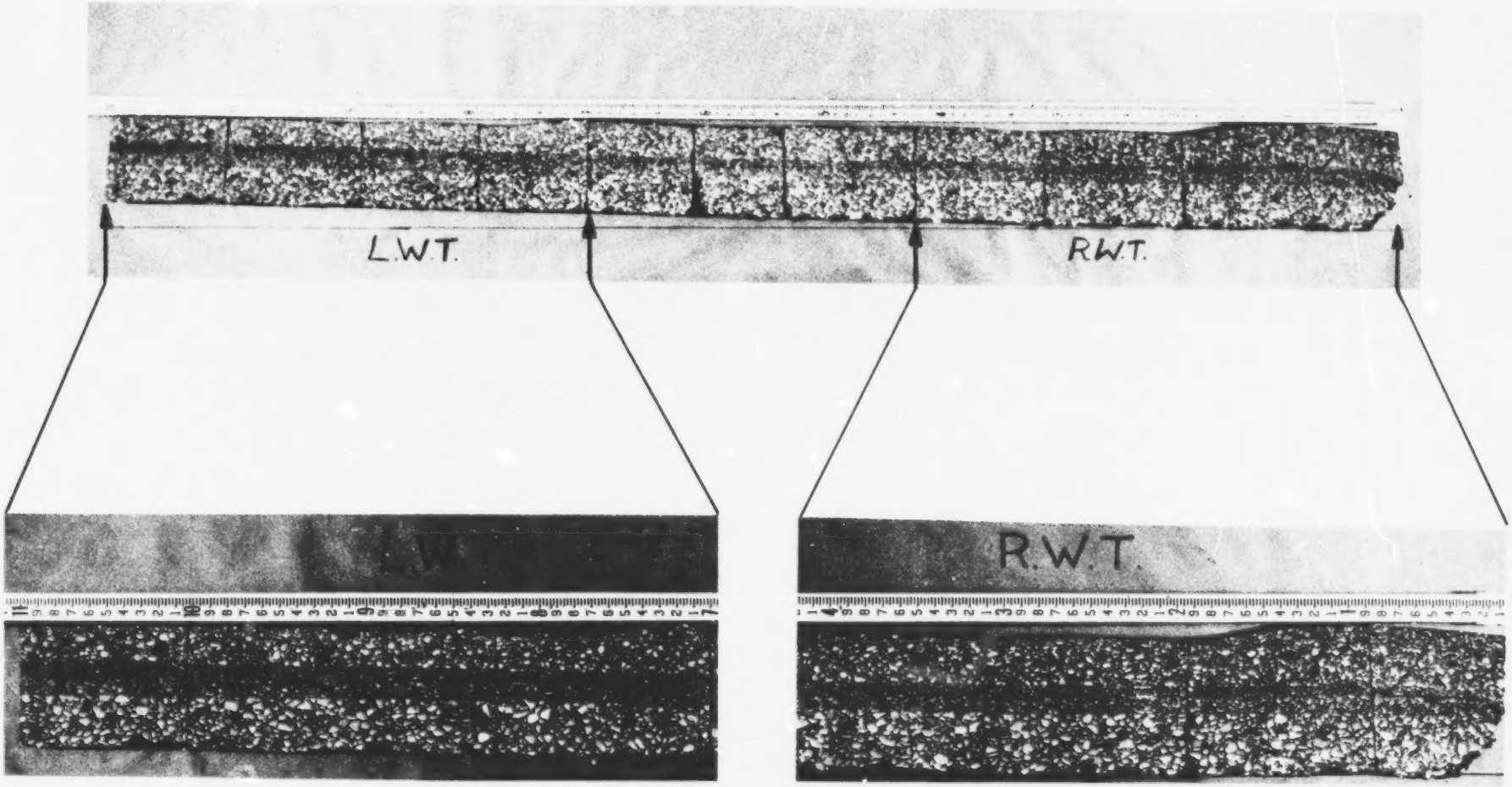
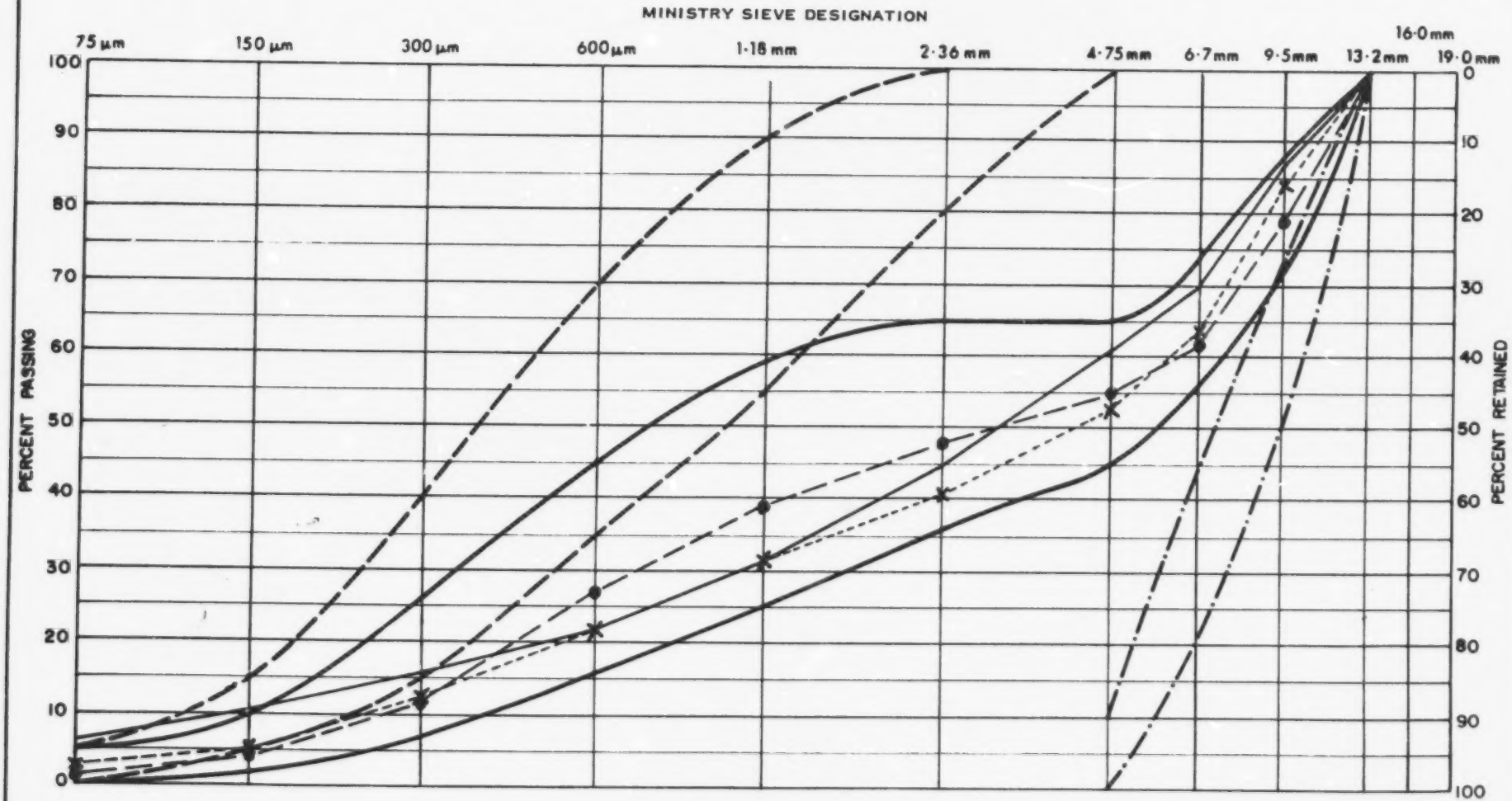


Fig. 11: Layer Thickness of Pavement Section

Ministry of Transportation and Communications
Ontario

SEMI—LOG GRADING CHART



x ----- x Core Samples

●——● Mix Design

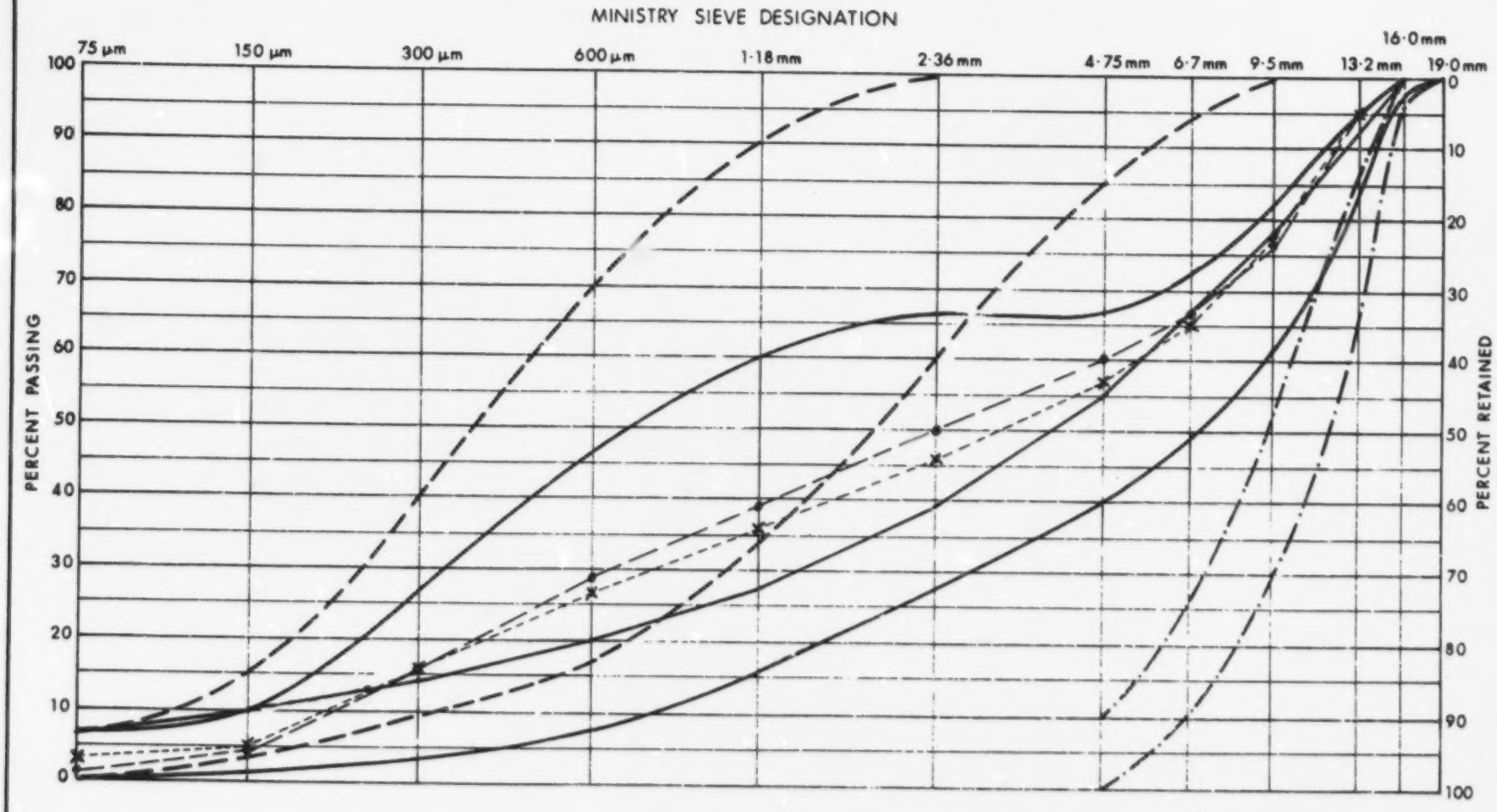
——— Fuller Maximum Density Curve

H.L.-1 & 3

Fig. 12

Ministry of Transportation and Communications
Ontario

SEMI - LOG GRADING CHART



H.L. - 4

Fig. 13

PR-CC-346 81-01 (FORMERLY 08-MT-98)

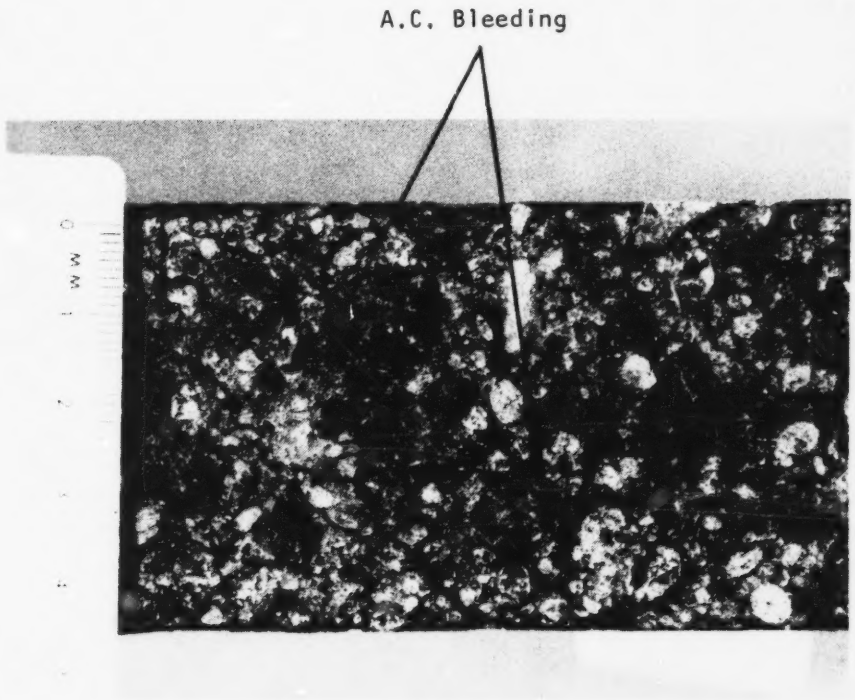


Fig. 14: Close-Up of Surface,
Showing A.C. Bleeding

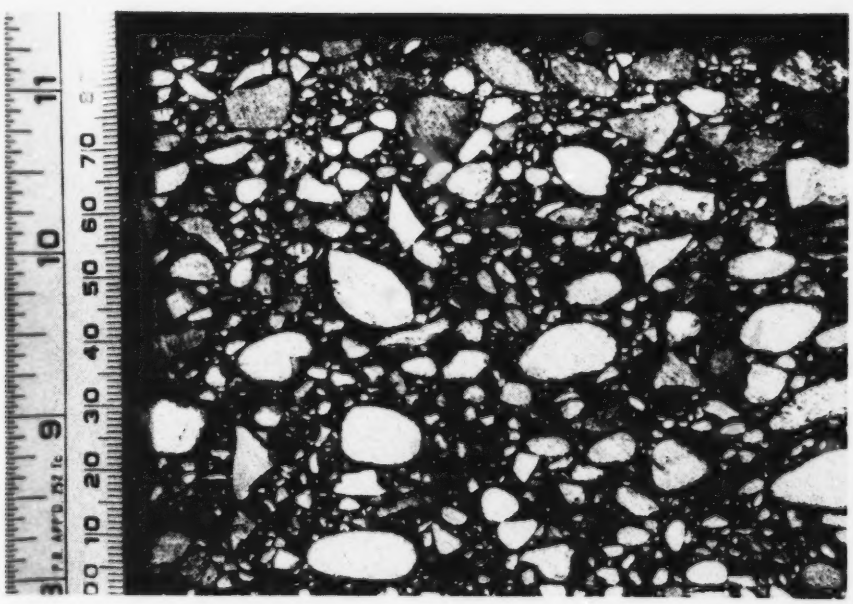


Fig. 15: Close-Up of Surface Course HL-1
and Binder Course HL-4

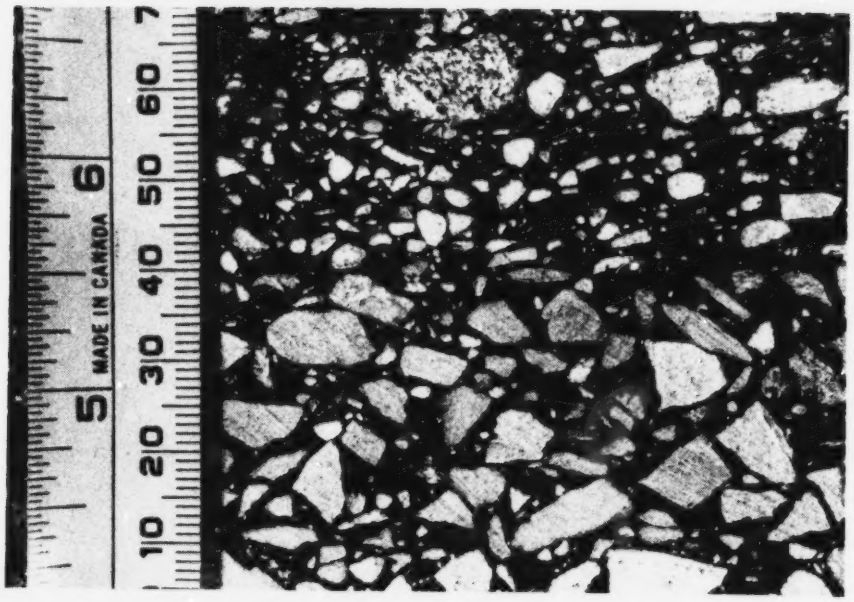


Fig. 16: Close-up of old HL-1 surface course

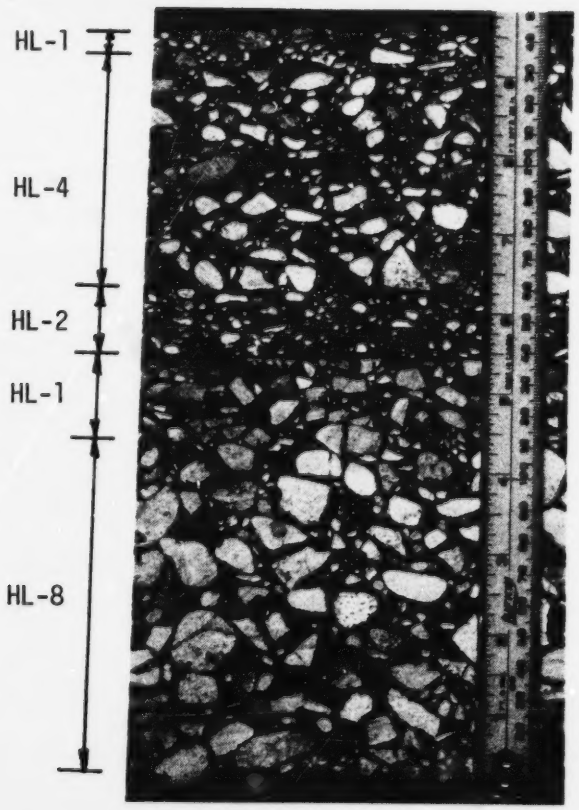


Fig. 17: Different layers of the pavement

APPENDIX 1

Soil Investigation Report

memorandum

Ontario

To: Mr. D. Lynch,
Head, Bituminous Section,
Engineering Materials Office.

Date: 82 10 18

Attn: Mr. K. Tam.

From: Soils & Aggregates Section.

Re: Pavement Rutting on Hwy. 401
Between Hwy. 28 and Hwy. 45

At your request the granular base and subbase in the area of the pavement distortion noted above was investigated to ascertain if the distortion could be attributed to difficiencies in these materials or to subgrade conditions. The investigation consisted of a field test hole and laboratory tests. Our findings together with our conclusions are given below.

Test Pits

On 82 09 22 the Bituminous Section cut a trench, about 0.5 m wide, directly across the east bound driving lane of Hwy. 401 at a location about 4 km east of Hwy. 28 where the pavement distortion was judged to be the most severe. After removal of the asphaltic concrete layers the Soils and Aggregates Section excavated a test hole located about 2 m left of the edge of pavement.

The test hole was carried out to a depth of about 0.3 m below the top of the granular base course layer. No seepage was observed during the course of the excavation. The base course consisted of a 100 mm thick layer of crushed gravel in a dry condition and was assessed to be in a dense to very dense state. The subbase course consisted of a pit run granular material (classified visually as a clear fine sand with gravel and with cobbles up to 100 mm in size) in a dry condition and was assessed to be in a dense to very dense state. The subbase course layer was proven to a thickness of at least 200 mm.

Laboratory Tests

Samples of the base and subbase course material were obtained during the field excavations. The results of laboratory testing are summarized below.

RECEIVED
OCT 18 1982
BITUMINOUS SECTION

Field Moisture Content:

Base Course Material	- 3.5%
Subbase Course Material	- 2.1%

Proctor Test on Base Course Material:

Maximum Wet Density	2.46	t/m^3
Maximum Dry Density	2.30	t/m^3
Optimum Moisture Content	6.8%	

Gradation Test on Base Course Material:

Sieve	Test Result % Passing	M.T.C. Spec. % Passing
37.5 mm	99.6	100
26.5 mm	96.5	
22.4 mm	94.6	
19 mm	91.4	70 - 100
16 mm	84.8	
13.2 mm	78.6	
9.5 mm	66.9	50 - 73
6.7 mm	58.4	
4.75 mm	51.0	35 - 55
2.36 mm	40.9	
1.18 mm	33.0	15 - 40
600 µm	27.0	
300 µm	18.2	5 - 22
150 µm	10.3	
75 µm	6.7	2 - 8

Percentage Crushed Particles Test on Base Course Material

Test Result %	M.T.C. Spec. %
39	50 minimum

California Bearing Ratio Test on Base Course Material:

At 0.1 in penetration	72%
At 0.2 in penetration	82%

The results of the field moisture content test confirms that the base and subbase course materials are in a relatively dry state.

Gradation test data indicates that the base course material conforms to the revelant M.T.C. Specification.

The percentage of crushed particles for base course material is specified as 50% minimum according to current M.T.C. requirements of Form 1010. Testing of the base course material indicated substandard material with only 39% crushed. Deficiencies in crushed particles will result in somewhat reduced load deformation properties; hence, considering the precision of the percentage crushed particle test, load - deformation testing was considered necessary to adequately evaluate the material.

The California Bearing Ratio Test was performed in order to assess the load deformation characteristics of the base course material. The C.B.R. value was found to be 82% based on the standard compaction effort (5.5 lb hammer, 12 inch drop) at the 0.2 inch (5 mm) penetration limit. The M.T.C. pavement design procedures do not require C.B.R. tests, but some agencies specify acceptable limits. The National Crushed Stone Association's flexible Pavement Design Guide specified that the C.B.R. of the base course shall be at least 100% based on the modified compaction effort (a 10 lb hammer, 18 inch drop). According to Yoder's text book on the Principles of Pavement Design, a minimum C.B.R. value of the base course should be 80% (standard compaction).

Conclusions

From the test hole observations the physical properties of the granular subbase course appear adequate. Furthermore, no subgrade moisture or adverse ground water conditions were encountered.

Based on the laboratory test results the granular base course material is substandard; however, considering the total thickness of pavement layers, it is our opinion that the pavement rutting has resulted from causes other than deficiencies in the base course material.

If you require comment or elaboration, please contact us.

M. MacLean

MDM/mm
c.c. B. McMaster

M. D. MacLean,
Soils & Aggregates Engineer.

APPENDIX 2

- Bituminous Mix Design Results for HL-1 and HL-4 of Contract 74-167

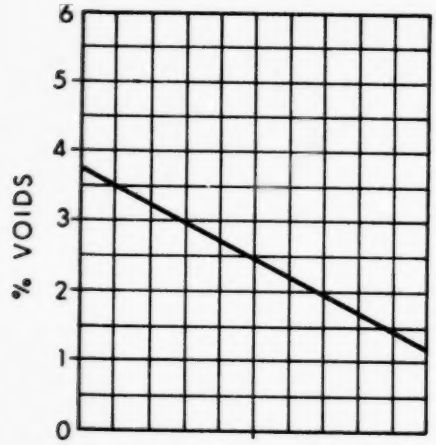
- Memorandum from R.P. Northwood to W.R. Bennett on Low Stability and V.M.A. Mixes

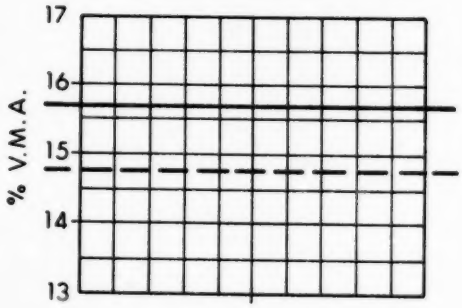
BITUMINOUS MIX DESIGN RESULTS

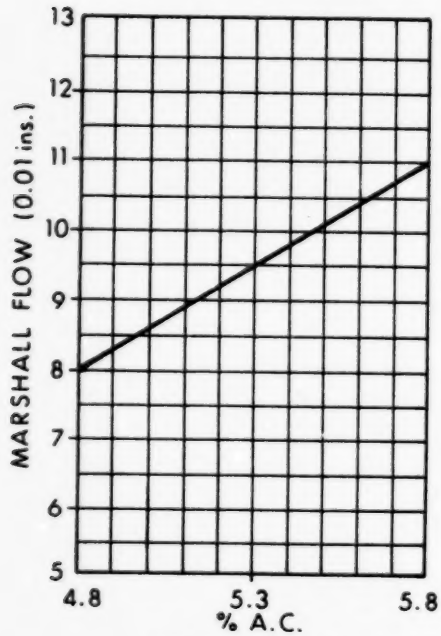
CONTRACT NO. 74-167 HWY. NO. 401

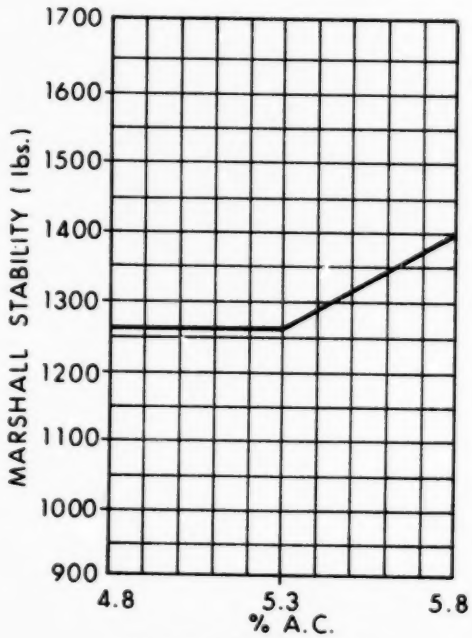
LOCATION HWY. 28 E'LY. TO HWY. 45, 5.9 MILES

MIX HL 1, 56 % SAND = 55 % PASS # 4









LAB. MIXES CHOSEN

	NO.1	NO.2	NO.3
% SAND	56		
% PASS # 4	55		
% A.C.	5.2		
% VOIDS	2.7		
FLOW	9.2		
STABILITY	1270		
① % V.M.A.	15.7		
② % V.M.A.	14.8		

1. INCLUDES A.C. ABSORPTION
2. EXCLUDES A.C. ABSORPTION

FIELD MIXES CHOSEN

	NO.1	NO.2	NO.3
% RET. # 4	45		
% PASS # 4	55		
% A.C.	5.2		

MATERIAL SOURCES

FINE AGG. # 1 *F. NELSON SCR. SAND # 2*

FINE AGG. # 2

COARSE AGG. *HAVELOCK*

ASPHALT *GULF MONTREAL 85/100*

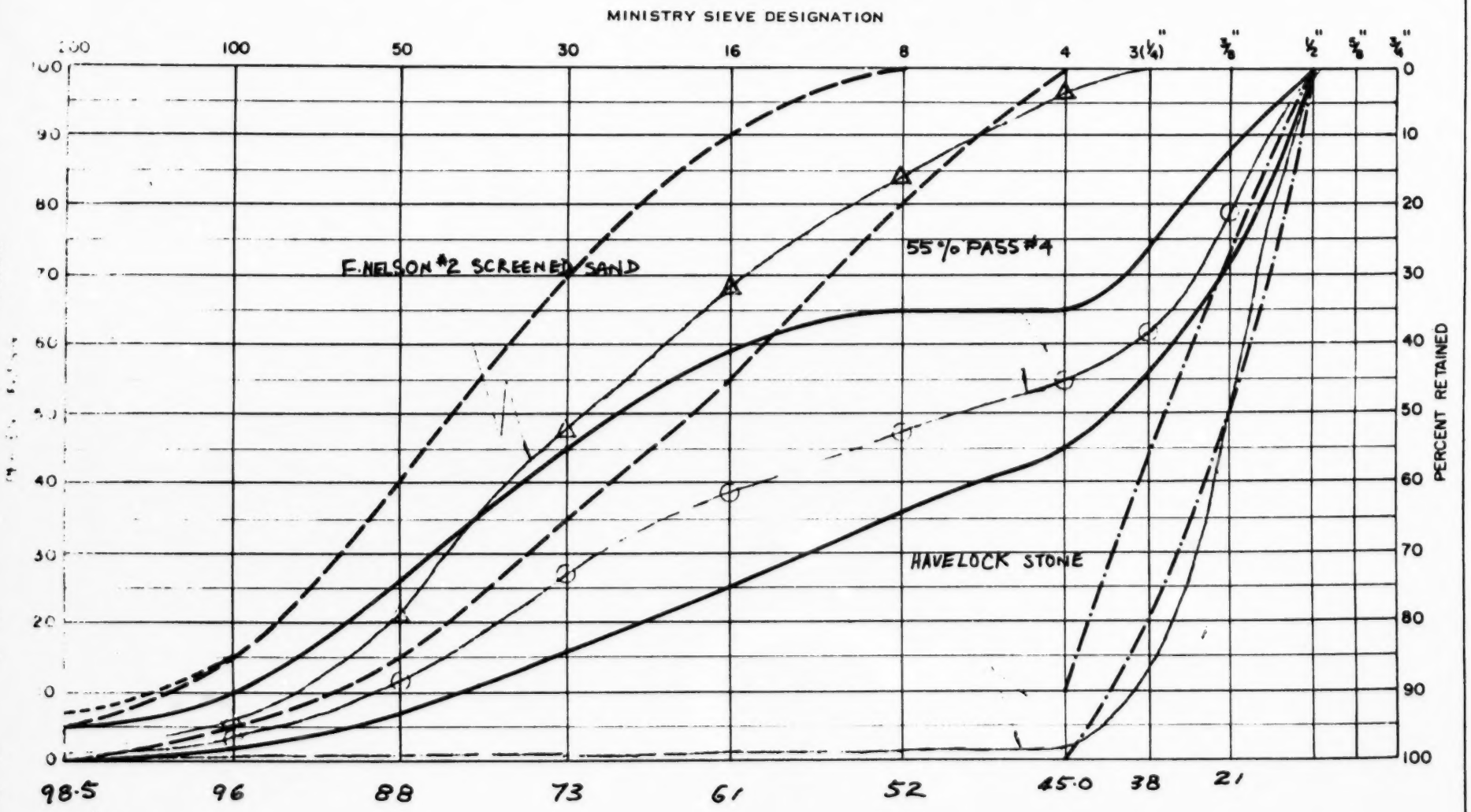
REMARKS *USING NELSON SCREENED SAND # 2 AS FINE AGGREGATE. THIS IS NOT SATISFACTORY HL-1. MIX DUE TO LOW STABILITY & BORDERLINE V.M.A. BUT IT IS THE BEST MIX FOR MATERIALS AVAILABLE*

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO

74-167
HL-1

7.

SEMI—LOG GRADING CHART



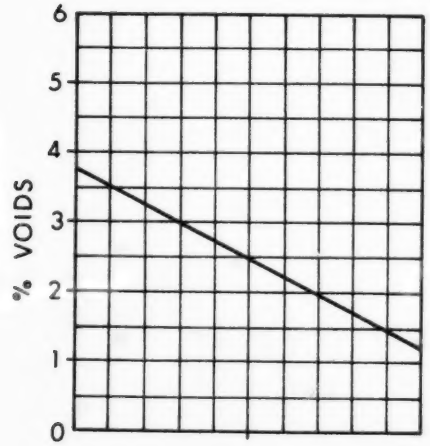
H.L.-1 & 3

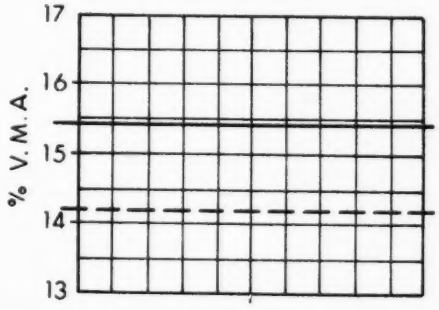
BITUMINOUS MIX DESIGN RESULTS

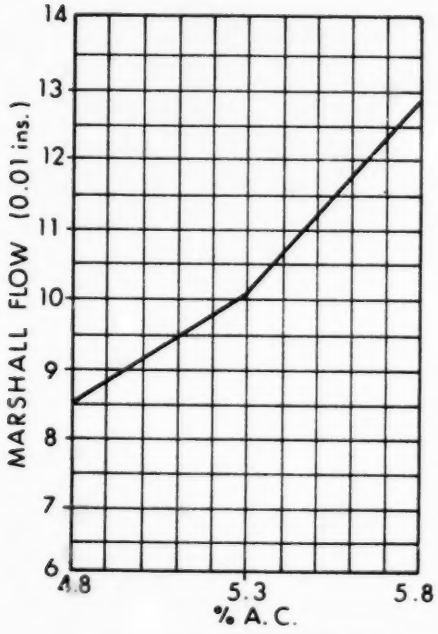
CONTRACT NO. 74 - 167 HWY. NO. 401

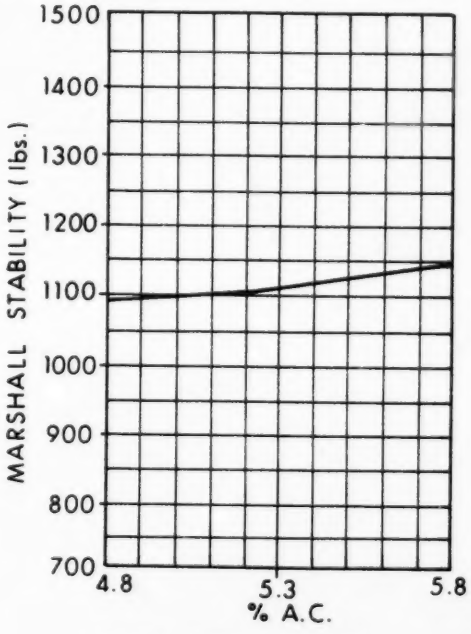
LOCATION HWY. 28 E'LY. TO HWY. 45

MIX HL 4, 59 % SAND = 60 % PASS # 4









LAB. MIXES CHOSEN

	NO.1	NO.2	NO.3
% SAND	59		
% PASS # 4	60		
% A.C.	5.1		
% VOIDS	3.0		
FLOW	9.5		
STABILITY	1100		
① % V.M.A.	15.4		
② % V.M.A.	14.2		

1. INCLUDES A.C. ABSORPTION
2. EXCLUDES A.C. ABSORPTION

FIELD MIXES CHOSEN

	NO.1	NO.2	NO.3
% RET. # 4	40		
% PASS # 4	60		
% A.C.	5.1		

MATERIAL SOURCES

FINE AGG. # 1 *F. NELSON SCR. SAND*

FINE AGG # 2

COARSE AGG. *F. NELSON*

ASPHALT *GULF MONTREAL 85/100*

REMARKS *MIX GIVEN - USING NELSON SCREENED SAND AS FINE AGGREGATE*
THIS IS NOT A SATISFACTORY MIX DUE TO LOW STABILITY AND V.M.A.

JUNE 1966

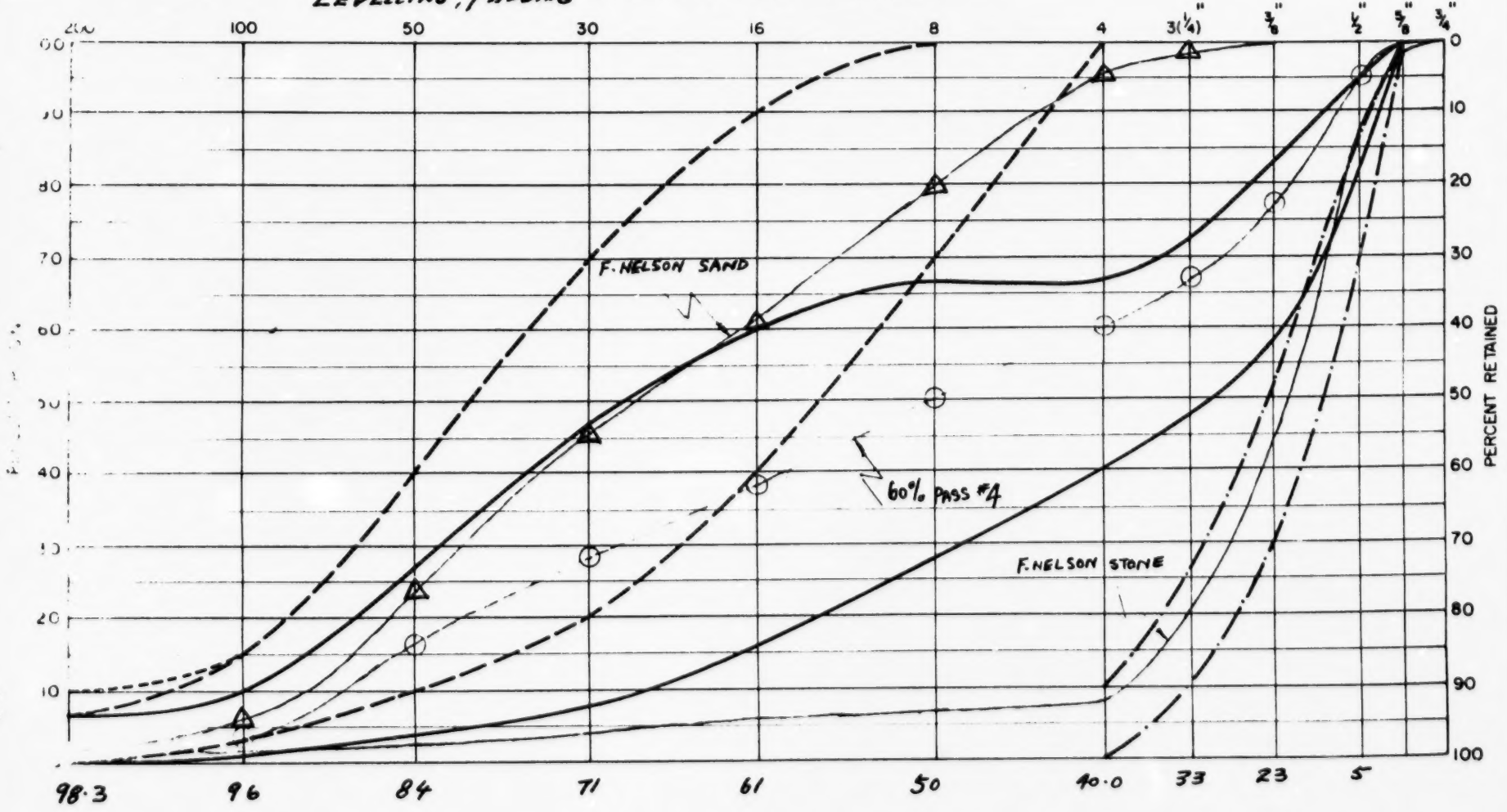
MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO

#7

SEMI—LOG GRADING CHART

74-167
HL-4 D
LEVELLING; PADDING

DEPARTMENT SIEVE DESIGNATION



H.L.—4 & 5

Memorandum

Mr. W.H. Bennett Quality Assurance Section West Building	From: Quality Assurance Section Construction Office Central Region
Attention:	Date: 1977 11 04
Our File Ref:	In Reply to:

Subject: Bituminous Mixes with Low Stability or V.M.A.

Following our discussions last week I asked Don Underwood and Peter Ksenych to select some mix design data from our files where, in the last 5 years, we have used mixes which were below our minimum requirements, in order to utilize available materials. They gave me about 20 examples and I have selected the attached 5 to send you.

Having made the point that we have not departed from what has been common practice in the past, I agree with you that we should not make major deviations from our established V.M.A. and stability criteria in the future. We will endeavor to select mixes which are in the Ministry's best interests from the performance and cost point of view and would welcome your comments if you think we are deviating too far from established criteria.

RPN:ja

R.P. Northwood
Head, Quality Assurance

c.c. K. Carter
R. Black
P. Ksenych
D. Underwood

APPENDIX 3

Surface Condition Survey

On 82 09 16, a general surface condition survey was made on eastbound Hwy. 401 from Liverpool Side Road Interchange to Wooler Road. The total distance surveyed was 124 km [77 miles]. The observations recorded herein are for the driving lane only. Based on these observations, some 37 km (23 miles) of road is affected by medium to severe rutting. Rutting is very minor in the eastbound passing lane and in the westbound lanes.

HWY. 401: SURFACE CONDITION SURVEY

(N = None, SL = Slight, M = Medium, SE = Severe, W.T. = Wheel Track)

Highway Mileage (0 at Hwy. 11 Interchange)		Road or Highway	Surface Rutting Condition				REMARKS
(km)	Dist. (km)		N	SL	M	SE	
28.3		Liverpool Rd.			X		
30.1	1.8	Brock Rd.		X			
31.8	1.7	Church St.		X			None at weigh scales
37.3	5.5	Weigh Scales	X				
43.1	5.8	Thickson Rd.	X				Cont. 82-03, OFC
67.8	24.7	Highway 115	X				
75.8	8.0	Morgans Cor. Rd.			X		
76.2	0.4	--				X	
79.3	3.1	Newtonville Rd.	X				
82.5	3.2	--				X	None at Service Sta.
88.1	5.6	Wesleyville Rd.		X			Also with W.T. Cracks
97.4	9.3	Hamilton-Hope Boundary Rd.				X	Section under investigation
101.3	3.9	--			X		
103.9	2.6	Burham St.			X		
114.1	10.2	Eagleson Rd.		X			None in places
118.3	4.2	Aird St.			X		
126.0	7.7	--	X				
126.5	0.5	--			X		
128.2	1.7	Percy St.	X				End of survey at Wooler Rd.
152.6	24.4						
		Σ (km)	66.6	20.7	24.4	12.6	Total = 124.3
		% of Total	53	17	20	10	Total = 100%